

SYSTEM AND METHOD FOR ANALYZING STRUCTURES SUBJECTED TO
CATASTROPHIC EVENTS

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Field of the Invention

The present invention relates to structure analysis systems, and in particular to a system for analyzing structures subjected to catastrophic events such as earthquakes, hurricanes, tornadoes, and man-made hazards.

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Background of the Invention

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Recent earthquakes in California (1989 Loma Prieta, 1994 Northridge), Japan (1995 Kobe), Turkey (1999), and Taiwan (1999) have clearly identified the vulnerability of structures to earthquakes and the staggering monetary losses due to such events. Losses from the Northridge earthquake alone are estimated at \$ 15 billion. Kobe earthquake losses are estimated at hundreds of billions of dollars.

The West Coast of the U.S. and the Pacific Northwest States are all susceptible to earthquakes. Discovery of the New Madrid fault poses a great danger to the Midwest region of the U.S.

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The Eastern Coast of the U.S. from Florida to Rhode Island is also susceptible to hurricanes. The hurricane season typically lasts from June through November each year. On an average, 12 to 14 hurricanes are generated in the Gulf each year. Losses from these hurricanes are also estimated at hundreds of billions of dollars. Losses from hurricane Andrew alone are estimated at \$ 25 billion.

Although all structures built in these regions are designed according to the national, regional and local building codes, there are catastrophic destructions and failures in these events. To understand why, the
5 building codes need to be analyzed.

Building seismic design forces are customarily provided by the Uniform Building Code (UBC). The UBC is updated from time to time with 1997 UBC being the current version in effect. The UBC states "The purpose
10 of the earthquake provisions herein is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function." The Structural Engineers Association of California (SEAOC) 1996 commentary adds the following to the UBC statement:
15 ".... or provide for easy repair."

The basic design procedure recommended by the Code assumes that the structure will undergo inelastic behavior and will sustain damage, i.e., may be permanently deformed or broken, during a design level
20 earthquake. This is implied by the use of the R-factor in the 1997 UBC, (i.e., "numerical coefficient representative of the inherent overstrength and global ductility capacity of lateral-force-resisting systems") to reduce the design lateral forces on a structure. A
25 typical design procedure is as follows: 1. selection of a design level earthquake intensity; 2. reduction of the applied forces (e.g., base shear) computed from the design earthquake by a Code recommended R-factor; and 3.
30 linear elastic analyses) for these reduced force levels to ensure elastic response such that the structure assumes its original shape after loading.

For different structure types the maximum R-factors are recommended in the Code. However, the selection of an appropriate factor for the structure under consideration, up to the maximum allowable Code value, is left to the discretion of the designer. Selection of the R-factor is usually determined by the performance criteria the owner wishes to establish. Thus, if the owner wishes the structure to be undamaged for the design level earthquake forces, the designer would select a value of R equal to 1.0. This decision, however, would result in a considerable increase in the cost of the structure and, given the random nature of the earthquake occurrence, this choice is not usually considered to be cost-effective.

A value of the reduction factor, R, greater than approximately 1.5 implies that the system will undergo inelastic behavior and will be damaged if a ground motion of design intensity is observed at the site. The coefficient of 1.5 represents the average factor, which is used in design to either factor the loads in load factor design or factor the yield strength of materials in working stress design. The R-factor is intended to refer to an acceptable level of damage via a global ductility response measure.

Thus, for a working stress design, if a R-factor of 10 is used in the design, the structure is assumed to sustain a global ductility of up to approximately $10/1.5 = 6.7$. Global ductility is a measure of damage. Typically for a building subjected to earthquake motions, it is defined as the ratio of the maximum building roof displacement and the roof displacement at which the first significant damage occurs anywhere in the building. Such assumed ductilities used in design

can only be confirmed by a nonlinear analysis (or experimental testing).

It has been widely published in literature that the methodology called "nonlinear analyses, nonlinear dynamics or failure dynamics" utilized in computer programs is the only realistic way to assess damage due to catastrophic events. Over the past 25 years, numerous reports from reputable Universities like U.C. Berkeley, Stanford and others have clearly stated these findings. The oil industry realized the value of such methodology and has incorporated it as a requirement in the API RP 2A Design Code. All offshore structures designed and built in the U.S. must comply with this Code.

Unlike the oil industry's design code, however, the building codes do not require such a state-of-the-art nonlinear analysis to confirm that the assumed global ductilities can be achieved in the adopted design. There are three primary reasons for not enforcing such a requirement in the building codes. First, non-linear analysis is too complex and too expensive to develop and validate for a wide variety of applications. It is important to note that the computational intensity of these algorithms have historically required Cyber mainframe class of computers available through places such as the Lawrence Livermore Laboratory. Second, it requires extensive manual intervention of an engineer with specialized training and theoretical background to set-up input models for a given structure. Third, because typical outputs from these analyses are voluminous, results interpretation is time consuming and requires specialized engineering knowledge.

Summary of the Invention

According to the principles of the present invention, a computer-implemented system and method for analyzing a structure subjected to a catastrophic event are provided. Based on linear elastic input data, the system automatically determines non-linear hysteretic behaviors without extensive manual intervention. The nonlinear models are then analyzed through any one of a number of nonlinear analysis techniques. Processing load of a computer is substantially reduced through various mathematical techniques which allow an ordinary server or workstation computer to conduct the analysis without requiring the power of a mainframe or super computer.

Brief Description of the Drawings

FIG. 1 is a functional block diagram of a system for analyzing structures according to an exemplary embodiment of the present invention.

FIG. 2 is a flow diagram of a method of performing a nonlinear analysis.

FIG. 3 is a graph of an exemplary cyclic degradation/deterioration in strength and stiffness of a truss-type post-buckling type element.

FIG. 4 is a graph of an exemplary envelope curve of a beam-column element.

FIG. 5 is a graph of an exemplary failure behavior of a beam-column element.

FIG. 6 is a diagram of an exemplary built-up section.

FIG. 7 is an exemplary summary of an inelastic sequence of events for elements of a structure.

FIG. 8 is an exemplary graphical image of a structure depicting different levels of damage for different members of the structure.

FIG. 9 is a plot of the number of damaged members as a function of a spectral acceleration from three different exemplary earthquake records.

FIG. 10 is a flow diagram of a method of rating a structure for risk assessment.

FIG. 11 is a flow diagram of a method determining damage functions.

Detailed Description of the Invention

As illustrated in FIG. 1, a structure analysis program 114 is maintained in an exemplary computer system 100, such as a WINDOWS-based or UNIX-based personal computer, server, workstation or a mainframe. The system 100 is connected to the Internet through, for example, an I/O interface 102, such as for a LAN, WAN, fiber optic or cable link, which receives information from and sends information to Internet users. While the system 100 is shown as a one computer unit for purposes of clarity, persons of ordinary skill in the art will appreciate that the system may comprise a group of servers depending on the load and database size.

The system 100 includes, for example, memory storage 104, processor (CPU) 106, program storage 108, and data storage 110, all commonly connected to each other through a bus 112. The program storage 108 stores, among others, a structure analysis program 114 containing all or portions of the routines 150, 200 and 300. The data storage 110 stores such data as physical properties of various members of structures,

mathematical representations of algorithms, damage
functions database and the like. Any of the program
modules in the program storage 108 and data from the
data storage 110 are transferred to the memory 104 as
5 needed and is executed by the processor 106.

The structure analysis system 100 of the present
invention includes simplified input data to the computer
programs used in the industry on large-scale projects.
Seismic retrofit of the Golden Gate Bridge is one
10 example of a large-scale project. The present
invention simplifies input data to an extent that the
input data structure mimics the typical linear elastic
analyses data structure.

Typically, for a linear elastic analysis of a
15 structure, structure data input (linear elastic input
data) includes the geometry data for each element or
member in the structure. As an example, for a steel
building, the linear elastic input data would define the
geometry of columns and beams in terms of their section
20 sizes, i.e., W12x24, etc. This data is sufficient to
calculate the stiffness and mass of the member. Given
the spatial distribution of members within a building,
the total structure stiffness and mass can be
calculated.

25 For nonlinear dynamic time history analyses,
complex nonlinear hysteretic behavior of members needs
to be specified. The member behavior may be different
even though the geometry section may be the same. For
example, a W12x24 section 3 feet long has dramatically
30 different nonlinear hysteretic behavior than a similar
member that is 15 feet long. Hence, input data for
nonlinear analyses becomes very complex and often

requires an engineering consultant with special knowledge in structure analysis.

The present invention overcomes this problem by having the user define the member inputs for the linear elastic model, i.e., element geometry is defined via section size specification. Additional linear input data required are the material type, and its yield stress to calculate overstress ratios. With the linear elastic input data, the program automatically calculates the complex hysteretic behavior internally.

The engineering community is familiar with "linear elastic" input data structures. In addition, translators and filters are available which further simplify the creation of input data. Moreover, according the principles of the present invention, new efficient solution techniques allow nonlinear analysis to be conducted by ordinary desktop workstations.

For example, nonlinear dynamic time history analyses require numerical solution of a large system of equations during each step of integration. For an earthquake time history analyses, the number of steps may vary from 1000 to 5000. For wave, wind and other natural forces, the number of steps may vary from a few hundred to a few thousand. Furthermore, within each time step, more solutions may be needed to achieve equilibrium, i.e., several iterations within a time step. The iterations may be due to element nonlinear behavior or large displacement effects or both. In contrast, linear analyses require such solution once only in dynamic analyses. Thus, if traditional numerical methods are utilized, they require tremendous computing power. The present invention overcomes this

problem by developing numerical schemes and methods to substantially reduce computation time required in solving the system of equations.

Once analyzed, the results are conveniently summarized in tabular and graphical form. The user also receives a color-coded picture of the structure in which damaged areas with varying damage levels are shown. A 3-D photo-realistic movie showing real-time dynamic responses may also be generated.

The structure analysis program 114 of the present invention is accessible through a network such as the Internet and is billed to a user on a pay-per-use basis. Thus, the user has no capital costs for hardware/software and pay-per-use is a convenient economical option for project expenses.

In another embodiment, the program 114 provides an engineering based structure rating system which identifies the potential risk due to catastrophic events for use by, for example, the financial institutions, engineering communities and insurance companies, corporate and other institutions to assess and manage their risks. Structures can be of any material and type (e.g., residential, non-residential, transportation infra-structure, etc.).

FIG. 2 illustrates a nonlinear analysis routine which is a part of the structure analysis program 114. Non-linear analysis types include, but are not limited to, time history analysis, static pushover, modal analyses, and fatigue analysis.

In step 202, linear elastic input data is received by the system 100. Typically, the linear elastic input

data for analyses includes: (a) geometry (nodal
coordinates, boundary conditions, nodal masses, etc);
(b) member connectivity/properties (section properties,
material properties, etc); (c) load data (earthquake,
5 wind, waves, currents, etc.).

The input data may be provided as an ASCII data
file conforming to the software's input specifications,
or it could be generated interactively through a
graphical user interface of the present invention. One
10 could also provide a scanned structure layout (e.g.,
layout from a property assessment report) and use this
layout in conjunction with the graphical user interface
to identify locations of structural elements and
generate an ASCII data file. Alternatively, filters in
15 the present invention can be used to convert data from a
suite of existing programs used in the industry to
generate an ASCII data file that conforms to the present
invention's software input specifications.

The linear input data to be received in step 202
20 includes global definitions of material properties,
section properties for stiffness calculations, section
properties for Building Codes and Other Code checks,
wave properties, miscellaneous data, group properties,
eccentricities and local fixed end forces. Access to
25 AISC and other standard rolled sections is achieved in
the global section library.

For linear elastic analyses, these items are
sufficient to perform the analysis. For nonlinear
analyses, however, item (b) above requires a much more
30 extensive definition of member data in terms of complex
nonlinear hysteretic member behavior.

Typically, nonlinear hysteretic member behavior (non-linear input data) comprises a definition of three components: (a) envelope behavior or curve; (b) cyclic degradation/deterioration in strength and stiffness (or simply degradation behavior); and (c) failure behavior. These components may be defined from databases and or lookup tables created from experimental observations, analytical formulae, empirical formulae, and/or any combination of the above. Interpolation and/or extrapolation formulae may be used wherever appropriate. In the present invention, all of the above methods are used wherever appropriate.

One of the more difficult tasks in any nonlinear analysis procedure is the selection of the appropriate element type to model a member in the structure. Once an element type is selected, the corresponding material, section and element properties are required to adequately define the physical behavior of that member. Selection of such properties can be tedious and requires considerable experience.

In step 204 of FIG. 2, complex member hysteretic behaviors are automatically generated. Specifically, step 204 automatically generates the nonlinear physical properties for various element types used to model tubular members, general AISC and other standard rolled sections, built-up sections, general sections, and user-defined sections. Materials may be steel, concrete, masonry, reinforced concrete, or user defined materials. Element types can be grouped as beam-column type, truss-type, foundation-type, and general/special type.

Thus if a truss-type post-buckling type element is selected to model tubular braces of an offshore structure, the user needs to specify only the member name, its connectivity, its diameter and thickness, its yield stress and the effective length factor as part of inputting the linear elastic input data in step 202. Step 204 of the routine 200 automatically generates the nonlinear envelope curve needed to define the member's physical behavior and selects an appropriate stiffness and strength deterioration algorithm for cyclic loads as shown in FIG. 3. The algorithms for generation of these properties have been derived and compiled from an extensive experimental database.

Similarly, if the 3-D nonlinear large displacement beam-column element with distributed plasticity was used to model a W-section representing a column of a building, then the user specifies this section in the global library. The nonlinear properties of the beam-column element are automatically calculated based on the W-section properties, the member connectivity and user defined yield material properties, the effective length factor as shown in FIG. 4. As an example, the in-plane and out-of-plane bending envelope quadrilinear curves are generated according to the following equations:

$$EIy1 = E * Iy \quad (1)$$

$$EIy2 = FAC1 * EIy1 \quad (2)$$

$$EIy3 = FAC2 * EIy2 \quad (3)$$

$$EIy4 = FAC3 * EIy3 \quad (4)$$

Where

E = Modulus of Elasticity

I_y = In-Plane moment of inertia

FAC1, FAC2, FAC3 are factors.

These factors are calculated based on the member's kl/r ratios and compactness, or experimental data or other sources. For example, for a tubular column, FAC1 may be 0.428, $FAC2 = 0.048/FAC1$ and $FAC3 = 0.001$.

$$M_{y1} = FAC4 * P_m \quad (5)$$

$$M_{y2} = FAC5 * P_m \quad (6)$$

$$M_{y3} = FAC6 * P_m \quad (7)$$

10 Where

M_{y1} , M_{y2} , M_{y3} are yield bending moments

P_m = Plastic moment based on yield stress and geometric properties and

FAC4, FAC5, FAC6 are factors.

15 These factors are calculated based on the member's kl/r ratios and compactness, or experimental data or other sources. For example, for a tubular column, FAC4 may be $\pi/4$, $FAC5 = 0.92$ and $FAC6 = 0.98$.

Note that similar calculations are performed for out-of-plane bending. Torsion behavior is assumed linear while the axial behavior may be calculated using the Load Factor Resistant Design (LRFD Buckling equation) or any other appropriate equation or experimental data.

25 Interaction amongst the two bending, one torsion and axial behavior is considered by a four-dimensional yield surface.

The failure behavior (third type of the non-linear input data) for the beam-column element may be defined

in terms of critical cumulative rotational ductility and critical cumulative axial ductility. In the example shown in FIG. 5, when the critical cumulative resultant rotational ductility from the in-plane and out-of-plane bending exceeds a threshold value, member failure is initiated. The subsequent failure behavior is characterized by the equation shown in FIG. 5.

Bridges, especially older steel bridges such as the Golden Gate Bridge of San Francisco, CA, comprise built-up sections. The present invention can handle almost any kind of a steel bridge built-up section. For a typical section shown in FIG. 6 comprising external and internal plates, external and internal angles, and lacing, the user specifies the information shown in the figure. Based on concepts similar to the beam-column described above, the present invention automatically calculates the nonlinear hysteretic bending and axial behavior.

For a masonry infill wall, the user needs to specify the panel's geometry (panel width, height, thickness), yield stress (to calculate overstress factors) and modulus of elasticity as part of the linear elastic data input. The panel hysteretic behavior is automatically defined as two diagonal struts. The strut properties are based on experimental and analytical data. Similar procedure defines properties of shear wall elements.

For reinforced concrete elements, the user specifies the concrete geometry and steel reinforcement geometry and layout as part of the linear elastic data input. Material properties for steel and concrete are input as stress-strain relations. Based on classical plasticity theory and other iterative numerical

procedures, the bending and axial nonlinear envelope behavior for a beam-column element type are automatically calculated. Cyclic degradation in strength and stiffness need only be defined by the user as light, moderate, and heavy. These are automatically translated to appropriate factors obtained from experimental and analytical data.

Foundation comprises the foundation elements and soil. For high-rise buildings in soft soil, the foundation elements may be piles driven in soil to appropriate depths. They may be steel, wood or concrete piles. The soil type characterizes soil elements. Here the soil may be specified by its type (e.g., clay, sand, sand etc.) and its other basic properties readily available from literature or specific soil boring laboratory tests. At the user-specified depths in the soil, the soil layer at the depth is modeled with three-orthogonal truss-type elements: two lateral and one axial. The nonlinear hysteretic behavior of each of these truss-type elements is automatically generated. In its present form, a coupled structure-foundation nonlinear analyses can be performed.

Loads may be static and/or dynamic. For static loads pushover analysis, the present invention in step 204 automatically generates static load pushover profiles for earthquake loads and wave/wind/current loads in addition to the nonlinear hysteretic member behaviors.

For earthquake loads, the user inputs the design response spectra for the three orthogonal directions with appropriate scale factors. Step 204 then automatically generates a static pushover profile by

performing the eigen solution, combining the modal forces from modes capturing up to at least 90% of the mass, and correcting the pushover profile to match the story shears and overturning moments.

5 For wave loads, the wave/current/wind is passed through the mathematical model and the snap-shot profile defining the largest base shear in the time history is selected as the load profile. The analysis is performed automatically where only the load analysis is performed
10 without the accompanying structural response calculations. Input for these loads is minimal and standard.

 Once all nonlinear data have been determined in step 204, analysis of the structure based on the derived
15 non-linear input data is performed in step 206. As discussed previously, for nonlinear dynamic time history analyses, the system of equations needs to be solved at every time step where there is a change of state in any member. If large displacement effects are included in
20 the analysis, it may be necessary to solve the system of equations more frequently. Solving the system of equations here implies reformation and reduction of the global stiffness matrix. Moreover, within the time step, iterative procedures must be utilized to achieve
25 equilibrium before proceeding to the next step. To minimize computation time the present invention optimizes the solution by (a) minimizing storage of the stiffness matrix utilizing minimization procedures such as bandwidth minimization, profile frontal method, etc.;
30 (b) utilizing numerical solution methods that require only one copy of the stiffness matrix; (c) utilizing storage and solution schemes where only the portion of stiffness matrix that has changed due to change in

member state and/or large displacement effects is re-formulated and reduced; (d) utilizing higher order numerical integration schemes which allow increase in step size and hence reduction in the total steps
5 required for analysis; and (e) utilizing hardware specific available accelerations. These novel features allow a drastic reduction in computational power necessary to conduct the non-linear analysis of structures.

10 For a static analysis, additional analysis automation is provided by a self-sensing static analysis capability. It allows users to perform ultimate capacity analyses accurately in a minimum amount of time. It has been established that the automatic (self-sensing) load
15 stepping procedure is less sensitive to the analysis approach used than the manual load stepping procedure.

The addition of a self-sensing (automatic) dynamic option allows the step size to be varied during a dynamic analysis. This option results in considerable
20 savings in computational and engineering costs. The criterion for time step control is based on severity of nonlinear response in any given time step. The unbalanced force vector at the end of the time step defines the severity of nonlinear behavior. If the
25 Euclidean norm of this unbalanced force vector exceeds a user specified tolerance, the step size is reduced by the user specified factor (commonly 0.5), and the step is repeated. Alternately, if the Euclidean norm of this unbalanced force vector is below the specified
30 tolerance, the subsequent step sizes are increased by a user-specified value (normally 2).

Once analysis is complete, the results are outputted and displayed in step 208. For static pushover or dynamic time history ultimate capacity analyses of large structures, it is time consuming to view,
5 interpret and digest the voluminous results produced. To ease the analyst's work, three options in the present invention are provided.

The first option is to print a summary of inelastic sequence of events for members, an exemplary summary of
10 which is shown in FIG. 7. The summary includes an event sequence number, static increment number and load step within the increment when the member becomes inelastic, the total load step or dynamic time step, member name experiencing the inelastic behavior, member type, member
15 group number, nodes I and J, D/C ratio, and group title. Each nonlinear member's maximum ductilities are also available in terms of resultant bending, axial tension, axial compression and axial total. First, a list in order of inelastic event sequence is available, followed
20 by the sorted list of members experiencing maximum to minimum damage. Further sorting is available for different member types. Other information may also be available.

A second option is to generate one or more of at
25 least two forms of graphical output: (a) a color-coded picture of the structure depicting all damaged members from maximum to minimum (a gray scale version of the color-coded picture is shown in FIG. 8); and (b) a color-coded animation of the structure response. The
30 color or the gray scale codes the damage in the braces summarized in FIG. 7. The upper panel braces LX1 and LX2 have maximum axial ductilities of approximately 15 and are shown in dark shade (red in actual application). The

lower panel braces LX3 and LX4 are in light shade (blue in actual application) since their maximum ductilities are approximately 4.0.

The present invention provides a mechanism to
5 quantify local, regional and global damage and maps these quantities to Building and Other Code Requirements. As an example, for a high-rise building subjected to a earthquake time history, the user needs to specify the following information only; (a) number of
10 stories in the building where damage is to be monitored; and (b) the nodes at each story which are used to calculate average story motions and displacements. With this information, the present invention automatically calculates global damage measure, regional damage
15 measure and local damage measure. Global damage measure is defined in terms of global displacement ductility. Regional and local damage is defined in terms of inter-story drifts, inter-story shears, number of damaged members, their maximum and cumulative ductilities, and
20 their number of equivalent full plastic cycles. Other measures may be included. The methods used to calculate these quantities are widely known and available in public literature, standard text books.

For risk assessment, the analysis step 206 is
25 repeated for multiple load conditions. For example, to perform a seismic risk assessment of high-rise building, the building is analyzed for a series of seismic loads representing different earthquake intensities. The selection of earthquakes and the analyses can be
30 extremely time consuming, cumbersome and complex. The present invention simplifies the process. A single input data stream can be utilized to perform multiple analyses using a single set of ground motions. Each execution

run scales the single set of ground motions by user defined scale factors. Usually the first scale factor is selected such that damage initiates in the structure when subjected to this scaled ground motion.

- 5 Alternately, the user may specify different target spectral accelerations. Again, the first target spectral acceleration is selected such that damage initiates in the structure when subjected to this scaled ground motion. Scale factors for ground motions are
- 10 automatically calculated to match the user specified target spectral accelerations.

- Finally, all the analyses results are automatically tabulated and graphically output with different options. An exemplary table/graph is shown in FIG. 9. The y-axis
- 15 shows the seismic spectral acceleration and the x-axis is the number of damaged members from three different earthquake records (record 1, record 2 and record 3). Other parameters can be mapped in to generate similar graphs. As an example, the spectral acceleration could
- 20 be mapped into a mean return period from a probabilistic seismic hazard analysis. The mean return period can be translated into probability (i.e., the reciprocal of mean return period is the probability of occurrence). Hence y-axis could now be probability of occurrence.
- 25 Similar procedures could be followed for other damage measures.

- In another embodiment of the present invention, the system 100 also provides a structure rating routine
- 150 for individuals, financial institutions, insurance
- 30 companies, real estate brokerage firms, and other entities as shown in FIG. 10. The financial institutions can use the rating system to assess their lending portfolio and risks. Similarly insurance companies can

structure (e.g., one, two-story wood frame), when it was built, information about any upgrades and the like. In step 153, using this information, the structure's fundamental period is determined by either utilizing
5 Building Code equations or other well-known methods. The reason for obtaining the structure's fundamental period will be discussed later herein in detail.

In step 154, the routine identifies all active faults around the structure to be rated. In step 156,
10 the routine identifies the fault(s) that likely cause the most damage to the structure with the earthquake magnitudes the identified fault(s) can generate and their probability. This information can be obtained from a seismic hazard analysis or publicly available
15 databases.

In step 158, spectral accelerations for the structure from the identified fault in step 156 are obtained. The seismic hazard analysis and the structure fundamental period(s) provide the information on
20 spectral accelerations for the structure under consideration.

Referring to FIG. 11, a damage function routine 300 will now be discussed. In step 302, the routine 300 classifies all structures according to a predetermined
25 structure types such as building code guidelines, FEMA guidelines or any rational means. In step 304, further classification (sub-classification) of the structures within each classified type is done based on fundamental structure periods. This way, all structures within a
30 classified structure type whose fundamental structure period falls within a predetermined range are considered to be similar structures for the purpose of structure

analysis. This novel feature of sub-classifying structure types based on fundamental structure periods allows (a) creation of fewer damage functions that are applicable to a larger population and (b) a rational basis for interpolation/extrapolation of damage functions between periods. Thus if there are two masonry buildings, one 10 foot high and the other 20 foot high, and the layout of these buildings is such that their fundamental vibration periods are the same or very close to each other within a very small range, then only one building needs to be analyzed for creation of damage functions which can then be applied to both buildings.

In step 306, a damage function for each sub-classified structure is determined in the following way to create a database of damage functions. The damage functions database is created by performing nonlinear analyses of typical structures built under typical building codes and typical building materials.

As an example, nonlinear models of houses built in different seismic zones, different soil conditions, and under different building codes are analyzed using nonlinear time history analyses. The house models are subjected to different earthquake intensities to establish what earthquake levels initiate damage, and what earthquake levels cause structural failure. For this purpose, preferably several sets of earthquake ground motion records are selected. Each set of ground motion records is appropriately scaled to simulate conditions from damage initiation to failure. In addition, the locations and propagation of damage is identified. The damage is quantified into three categories: (a) global damage; (b) regional damage and

(c) local damage. A typical damage function is shown in FIG. 12.

In FIG. 12, the structure type analyzed is a deck-isolated steel offshore jacket structure which was analyzed for three earthquake records: Joshua, Saratoga and Newhall. Each record was scaled by three values. A total of nine nonlinear dynamic time history analyses were performed. The damage measure monitored here was the relative displacements between the deck and jacket top. Damage functions for each of the earthquake records is shown as identified by the earthquake record name. The curve 11 is the mean structure damage curve obtained by performing a regression analysis on damage functions from each record. While any number of ground motions may be used to calculate the mean damage curves (from 1 to 5 or more), three sets of ground motions are sufficient to generate mean structure damage curves since the uncertainty associated with the ground motion is far greater than the uncertainty in the response calculations.

If necessary, uncertainty in the response can be quantified by varying key input parameters (e.g., yield stress) and repeating the nonlinear analyses for each earthquake record. This process results in several damage curves for each earthquake record. A regression analysis can be performed to obtain a mean damage curve for the particular earthquake record. Finally, as before, regression analysis on the mean damage curves for each record provides the mean damage function accounting for the uncertainty in the response.

Non-structural damage functions are mapped to structural damage functions. As an example, if the

inter-story drift in the structural damage function exceeds a threshold value, non-structural damage is initiated (i.e., glass breaking, water/gas pipes shearing, etc.). Similar damage functions (structural
5 and non-structural) are created for each structure type and material type in step 306.

Referring back to FIG. 10, step 160 retrieves from the damage functions database one damage function of the sub-classified structure that most closely corresponds
10 to the structure to be rated based on its fundamental structure period and material properties.

Damage to the property may be structural damage or non-structural damage. Structural damage is obtained in step 162 by mapping the spectral acceleration obtained
15 in step 158 against the damage function retrieved in step 160.

In step 164, the routine determines an overall rating for the property from a scale of 1 to 10. The rating considers structural damage (global, regional,
20 local) and non-structural damage information in conjunction with the hazard under consideration (e.g., earthquake). Financial and other information may also be considered.

The structure rating routine 150 described above is
25 applicable to any hazard which can be probabilistically quantified and can be applied to hurricane loadings, waves/wind/current for offshore structures, and the like. Similarly, damage function routine 300 is applicable to other hazards (e.g. hurricanes, waves,
30 wind, etc.).

From the foregoing, it will be appreciated that, although specific embodiments of the invention have been described herein for purposes of illustration, various modifications may be made without deviating from the spirit and scope of the invention. For example, while the embodiment disclosed illustrates the present invention in an Internet environment, persons of ordinary skill in the art will appreciate that the system can be implemented in any computer network environment including the Intranet, LAN, WAN or the like. Accordingly, the present invention is not limited except as by the appended claims.